Public Works & Capital Projects

RESOLUTION NO. 2016282

RE: AMENDING THE 2016 ADOPTED COUNTY BUDGET AS IT PERTAINS TO THE DEPARTMENT OF PUBLIC WORKS

Legislators PULVER, BORCHERT, TRUITT, and SAGLIANO offer the following and moves its adoption:

WHEREAS, the Commissioner of the Department of Public Works advises that the Fallkill Dam which is on County owned property identified as 246-260 Creek Road in the Town of Poughkeepsie (tax parcel 134689-6163-04-842250-0000) is in dire need of repair; and

WHEREAS, the New York State Department of Environmental Conservation ("NYSDEC") dam safety regulations require that certain work be completed in order to keep the dam in a state of good repair and to ensure public safety, and

WHEREAS, the Fallkill Dam is currently classified as an "Intermediate Hazard (Class B) and is required to have an Engineering Assessment conducted every ten (10) years in accordance with the NYSDEC Dam Safety Regulations 6 NYCRR Part 673, and

WHEREAS, the Department of Public Works in consultation with the NYSDEC has determined that the improvement project (1) constitutes a Type II action pursuant to 6 NYCRR Sections 617.5(c)(1), (c)2 and (c)(6) ("SEQRA"), and (2) will not have a significant adverse impact on the environment, and

WHEREAS, an Engineering Assessment was completed and the findings identified repairs and/or rehabilitation measures that are necessary for various existing features in order to bring the structure into compliance with the dam safety regulations, and

WHEREAS, the Commissioner advises that additional funding is necessary for the design, permitting and construction of the referenced repairs, and

WHEREAS, the appropriation of these funds are critical for the repair and rehabilitation of the Fallkill Dam located in the Town of Poughkeepsie, and

WHEREAS, it is necessary to amend the 2016 Adopted County Budget to provide additional funds to repair and rehabilitate the dam as required by the NYSDEC, now therefore, be it

RESOLVED, that the Commissioner of Finance is authorized, empowered and directed to amend the 2016 Adopted County Budget as follows:

APPROPRIATIONS – Increase

H0498.5020.3110	Other Structures	$425,000
A.9950.9000	Interfund Transfer	$425,000
		$850,000

REVENUES – Increase

A.9998.95110.87	Appropriated Reserve Capital	$425,000
H0498.5020.50310	Interfund Transfers	$425,000
		$850,000

CA-173-16
CAB/kvh/G-0188
11/10/16
Fiscal Impact: See attached statement

STATE OF NEW YORK
ss:
COUNTY OF DUTCHESS

This is to certify that I, the undersigned Clerk of the Legislature of the County of Dutchess have compared the foregoing resolution with the original resolution now on file in the office of said clerk, and which was adopted by said Legislature on the 8th day of December 2016, and that the same is a true and correct transcript of said original resolution and of the whole thereof.

IN WITNESS WHEREOF, I have hereunto set my hand and seal of said Legislature this 8th day of December 2016.

CAROLYN MORRIS, CLERK OF THE LEGISLATURE

FISCAL IMPACT STATEMENT

☐ NO FISCAL IMPACT PROJECTED

APPROPRIATION RESOLUTIONS

(To be completed by requesting department)

Total Current Year Cost $ 425,000

Total Current Year Revenue $ ______
and Source

Source of County Funds *(check one)*: ☐ Existing Appropriations, ☐ Contingency, ☐ Transfer of Existing Appropriations, ☐ Additional Appropriations, ☑ Other *(explain)*.

Identify Line Items(s):
Capital Reserve - A.9998.95110.87

Related Expenses: Amount $ ______
Nature/Reason:

Anticipated Savings to County: ______

Net County Cost (this year): ______
Over Five Years: ______

Additional Comments/Explanation:
Funding needed to repair the Fallkill Dam based on an assessment performed in 2015. Repairs are required by the New York State Department of Environmental Conservation in order to maintain the Fallkill Dam in a state of good repair to protect property and public safety. The total funding of $425,000 is requested from the Capital Reserve for this project.

Prepared by: Rachel Kashimer, Budget Office

Prepared On: 11/3/2016

Dul.pdf

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Dutchess County
DPW *ENGINEERING*

Memo

To: Fallkill Dam

From: Jeff Akins, P.E.

Date: August 18, 2016

Re: SEQR CLASSIFICATION
FALLKILL DAM REPAIR AND REHABILITATION PROJECT
COUNTY ROUTE 100 (a.k.a. CREEK ROAD)
TOWN OF POUGHKEEPSIE

The subject project consists of repairs and/or rehabilitation to the existing County-owned Fallkill Dam. The Fallkill Dam is located on a County owned parcel identified as 246-260 Creek Road (*tax parcel 134689-6163-04-842250-0000*) in the Town of Poughkeepsie. The Fallkill Dam is classified as an Intermediate Hazard (Class B) and is required to have an Engineering Assessment (EA) conducted once every ten years in accordance with NYSDEC Dam Safety Regulations 6 NYCRR Part 673. The County utilized a consulting firm (C.T. Male Associates) to conduct an EA for the structure with draft findings completed in January 2016. The draft EA identified repairs and/or rehabilitation measures needed for various existing features in order to bring the structure into compliance with dam safety regulations.

The County will seek funding for the design, permitting and construction of the noted repairs. Upon discussion with the consultant and Region 3 permit staff it is determined that proposed repair and/or rehabilitation construction can be classified as a TYPE II Action. Specifically the proposed work conforms to the following definitions listed in 6 NYCRR §617.5 (Type II Actions):

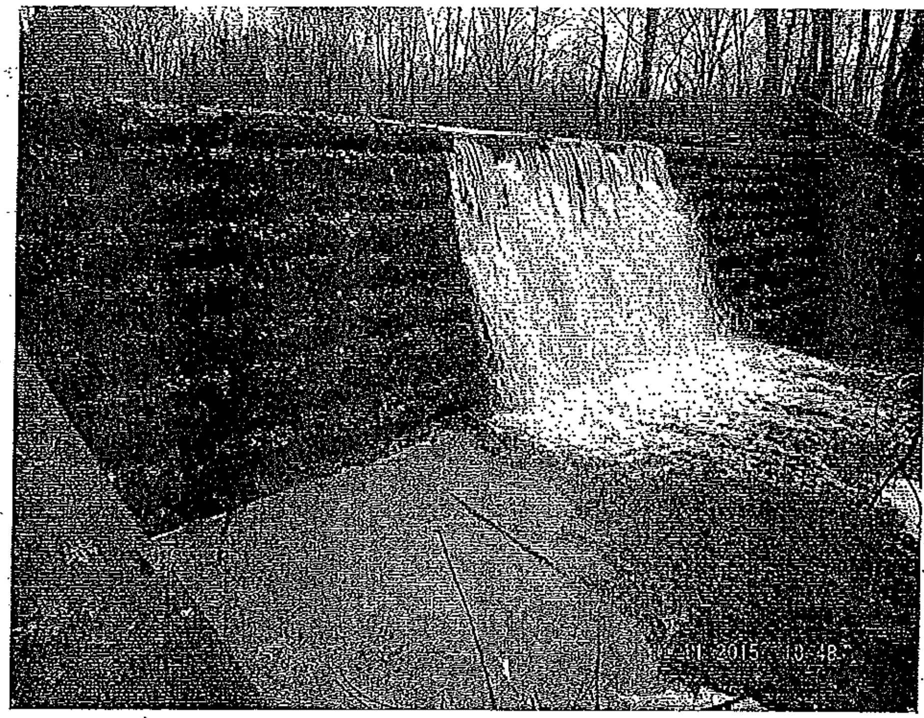
- (c)(1) – maintenance or repair involving no substantial changes in an existing structure or facility,
- (c)(2) – replacement, rehabilitation or reconstruction of a structure or facility, in kind, in the same site, including upgrading buildings to meet building or fire codes, unless such action meets or exceeds the thresholds in Section 617.4 of the Park,
- (c)(6) – maintenance of existing landscaping or natural growth.

The subject project is therefore classified as a SEQR Type II Action as per 6 NYCRR §617.5and no further action is required. The Dutchess County Department of Public Works is the SEQR Lead Agency for this action.

C.T. MALE ASSOCIATES
Engineering, Surveying, Architecture & Landscape Architecture, D.P.C.

ENGINEERING ASSESSMENT of FALL KILL DAM

NATIONAL DAM ID: NY01178
NY STATE ID: 212-0705



Owned and Operated by:

DUTCHESS COUNTY DEPARTMENT OF PUBLIC WORKS

DUTCHESS COUNTY, NEW YORK

Latitude N 41° 44′ 10″
Longitude W 73° 53′ 59″

Document Date:	January 15, 2015
Assessment Performed by:	Richard C. Wakeman, P.E. NYS License No. 057208

Engineering Assessment
Fall Kill Dam
Creek Road
Poughkeepsie, New York

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FIGURES

APPENDICES

1.0 INTRODUCTION

This Engineering Assessment of Fall Kill Dam in the City of Poughkeepsie, New York has been performed in accordance with the requirements set forth in the New York State Department of Environmental Conservation (DEC) Dam Safety Regulations, Part 673.13. Included in the preparation of this assessment were a review of existing documentation for the dam, the performance of an Initial Safety Inspection, a hydrologic and hydraulic analysis of the reservoir's watershed and dam, a hazard classification assessment of the dam, a stability analysis of the dam's earthen embankments, and a structural stability analysis of its concrete gravity section. Existing documentation reviewed included historic documents and photographs, past inspection reports, and the dam's Emergency Action Plan. On the basis of the findings of this assessment, conclusions and recommendations for bringing the dam into compliance with DEC regulations are provided. A recommended schedule for this work has also been developed.

2.0 INITIAL SAFETY INSPECTION

2.1 Document Review

Existing documentation in the possession of DEC was reviewed prior to preparation of this Engineering Assessment. Included amongst this information were:

- Historic documents pertaining to the dam, including letters, reports, and photographs;
- DEC inspection reports; and
- An Emergency Action Plan, prepared by the Dutchess County Department of Public Works, dated March 14, 2012.

2.2 Dam Description

Fall Kill Dam, formerly known as Hudson River State Hospital Dam, is located in the City of Poughkeepsie, New York, at latitude 41 degrees, 44 minutes, 10 seconds north and longitude 73 degrees, 53 minutes, 59 seconds west. The dam's spillway is a gravity section constructed of stacked stone masonry. Earthen closure embankments are present to each side of the spillway, the upstream sides of which are retained by vertical stone masonry walls. The closure embankments are present on the downstream side of these walls and are retained adjacent to the spillway by the spillway's abutment walls. A short distance north of the dam on its left side (looking downstream) is an earthen dike.

Fall Kill Dam impounds Fall Kill Creek to create Fall Kill Lake, a body of water which is identified by DEC as an unnamed pond. This pond was originally created for use as a raw water supply source for the Hudson River State Hospital, although it was abandoned for that purpose many years ago and is now reportedly used for recreational purposes only.

The earliest records recovered during the Document Review process date to 1911, where it is noted that the dam was present at the site. In an undated document, it is noted that a stone masonry dam was to be constructed on Fall Kill Creek to provide an "additional water supply." In this same document, it is noted that construction of the dam was substantially completed in November 1892. The dam was to be founded on sound bedrock at depths ranging from 3 to 7 feet below the original ground surface. It is stated that the dam's spillway was to be approximately 70 feet wide and approximately 17 feet high, with abutment walls extending 5 feet above the spillway's crest.

In 1921, extensive repairs were made to the dam which included facing the upstream side of the spillway with concrete, repointing of the dam's stone masonry, and the construction of a valve house. In addition, presumably to increase the reservoir's storage capacity, an earthen dike was constructed to the left of the dam, beginning north of a knoll present at the left end of the dam. Several photographs which were taken during performance of this work were recovered, and include one photograph which shows a concrete core wall within the earthen dike. No design or record information was recovered regarding any of the reconstruction work. Based upon the recovered photographs, however, the structure shown in 1921 appears to be relatively consistent with that currently present at the site. No other modifications or improvements are known to have been made to the dam, although at some point the dam's valve house was demolished.

According to DEC, the dam, consistent with the limited topographic survey performed for this assessment, is 220 feet long and 21 feet high as measured from its crest to its downstream toe. The spillway is identified by DEC as being 71 feet wide, which is relatively consistent with that determined from the field survey of 69.3 feet. Flow over the spillway discharges into Fall Kill Creek. From the limited topographic survey, the earthen dike has an approximate length of 135 feet and a maximum height of approximately 10 feet.

Copies of the historic drawings and documents pertaining to the dam are included in Appendix A. A copy of the limited topographic survey of Fall Kill Dam and the adjacent dike is included in Appendix B.

2.3 Physical Inspection

The physical condition of Fall Kill Dam was inspected on November 11, 2015, by personnel of our office. Weather conditions on the day of inspection were cool and rainy. At the time of inspection, the reservoir level was approximately 3 inches above the spillway crest. A follow up visit was made on December 30, 2015 to inspect the earthen dike.

Appendix C contains a Visual Inspection Checklist completed for the inspection and Appendix D contains photographs that are referenced on the Visual Inspection Checklist.

2.3.1 Earthen Embankments & Dike Section

Inspection of the dam began with its earthen embankments and the walls which retain the upstream side of each embankment. Only a portion of the upstream walls retaining the earthen embankment sections could be inspected due to the level of the reservoir at the time of inspection (approximately 6 feet below the top of the walls).

On the right side of the dam (looking downstream), the wall retaining the embankment was observed to be leaning several degrees from vertical (Photograph No. 5) for approximately 20 lineal feet of its length. Previous DEC inspections of the dam have noted similar conditions. The floor of a former valve house is located on the crest of the right embankment as shown in Photograph No. 2. Grades along the embankment crest were slightly lower than the floor of this former structure and appeared to vary by less than 6 inches in elevation. The inclination of the embankment's downstream slope was visually estimated to be 2.5:1 as shown in Photograph No. 6. Significant quantities of vegetation, including several large diameter trees and woody brush, are present on the embankment, also shown in Photograph No. 6.

On the left side of the spillway, concrete has been placed to cap the vertical stone masonry wall present on the upstream side of the embankment. As shown in Photograph No. 14, blocks of the underlying stone masonry are locally dislodged. . As shown in Photograph No. 21, a section of the wall's concrete surface had been eroded/removed to a depth of approximately one-half (1/2) inch. The crest of the embankment retained by this wall varies in elevation by as much as 18 inches and is lower than the wall at its interface with the original ground, which under periods of elevated reservoir levels could result in flow around the dam. This condition has been previously observed during DEC inspections of the dam. The inclination of the embankment's downstream slope was visually estimated to be 3:1 as shown in Photograph No. 7. Significant quantities of vegetation, including several large diameter trees and woody brush, are present across the embankment, also shown in Photograph No. 7.

As shown in Photograph Nos. 13 and 15, concrete has been placed over each of the spillway's stone masonry abutment/training walls. The depth of this concrete ranges from approximately 1 foot to as much as 6 feet. At several locations, the dam's concrete has been covered with graffiti (Photograph No. 22). As shown in Photograph No. 15, pieces of stone masonry have become dislodged or are missing.

Approximately 135 feet left of the dam is an earthen dike which is separated from the spillway section by a topographic high ("knoll") of natural ground. As seen in Photograph No. 1, a concrete core wall is approximately centered within the earthen dike. At the time of inspection, large portions of the core wall were observed to extend above adjacent embankment grades. The concrete used to construct the core wall appears to be cyclopean (Photograph No. 23) and has experienced moderate amounts of erosion and deterioration (spalling). In several locations, the concrete's aggregate has become exposed and is loose. Deterioration of the core wall's concrete has been observed during past DEC inspections of the dike.

In general, the dike is covered with numerous trees and woody brush (Photograph No. 3), conditions which have been noted during past DEC inspections. Progressing left to right along the dike, grades along its crest were visually estimated to vary by as much as 18 inches. The crest width varies significantly along its length, from approximately 10 feet to more than 30 feet in some locations. It was visually estimated that the inclination of the dike's upstream slope was approximately 1:2.5 (V: H), although at several locations, apparent erosion of the slope has resulted in local oversteepening of the same. Significant amounts of woody brush and small diameter trees are present along the upstream slope of the dike as shown in Photograph No. 4. Similar conditions are present along the downstream slope of the dike, where moderate amounts of woody brush and trees are present (Photograph No. 8). The inclination of the downstream slope was found to vary considerably along the length of the dike, from approximately 1:20 near its right end to 1:3 along its left half.

Seepage was noted emerging from the downstream toe of the dike at a point approximately 25 feet from the left end of the dike's core wall (Photograph No. 10). The rate of seepage was visually estimated to be on the order of 1 gallon per minute at the time of inspection. Areas of standing water were also present along the downstream toe of the dike, and the source of this water is unclear.

2.3.2 Spillway

The dam's spillway is constructed of stacked stone masonry, approximately 17 feet in height and 69.3 feet in width, as shown in Photograph No. 12. As noted during past DEC inspections of the dam and as shown in Photograph No. 16, apparent leakage through the spillway section was observed along its downstream face. Occasional pieces of deadfall were observed along the spillway crest (Photograph No. 17). Approximately 3 feet of standing water is present at the downstream toe of the spillway, where the possible remnants of a plunge pool are present as shown in Photograph No. 27. Little to no rip rap was observed within the area, and that which was present appeared to be of a size consistent with New York State Department of Transportation "light" stone filling. The downstream channel was observed to be relatively broad, although occasional deadfall was present within the channel (Photograph No. 11).

In general, the stone masonry comprising the spillway and its abutment/training walls was in fair condition. All stone surfaces have experienced slight to moderate amounts of weathering. However, little to no grout was present within any of the joints between pieces of stone masonry. As previously noted, several pieces of masonry were observed to be missing or displaced. Where the stone masonry was capped with concrete, the visible surfaces were generally in good condition, with little to moderate erosion/spalling typically noted. Occasional structural cracking of the concrete was observed, although no horizontal or vertical offsets were noted (Photograph No. 24).

Although several of the concrete construction joints were noted to be open, with gaps as great as one-half (1/2) inch as shown in Photograph No. 26, no loss of ground appeared to be occurring from behind the abutment/training walls of the spillway.

With the exception of the upper part of the concrete facing the upstream side of the spillway, no observations could be made of its condition below the reservoir

2.3.3 Piping & Valves

The remains of the former valve house, shown in Photograph No. 2, is present immediately downstream of the wall facing the upstream side of the right closure embankment. Two (2) large concrete slabs are present over its substructure. A wooden foldout ruler was lowered through a gap between the slabs and encountered soil at a depth of 1 foot beneath the top of the concrete slabs, the origins of which are unknown. Extending from the downstream toe of the right closure embankment to a point approximately 75 feet downstream of the same is a stacked stone masonry wall, which faces the downstream channel. Present just beyond this wall is a 42-inch diameter cast-iron pipe (Photograph No. 18) and a sluice gate (Photograph No. 19). This sluice gate does not appear to be operational, as no means for its control were evident. Slight leakage, estimated to be approximately 1 to 2 gallons per minute, was noted from the top left side of the sluice gate (Photograph No. 21). It has been noted during previous DEC inspections of the dam that this gate was cracked near the top left hand side and leakage had been observed emanating from that point.

3.0 HAZARD CLASSIFICATION

To assess the dam's hazard classification, failure of the dam was simulated and impacts to downstream roadways and structures determined following the guidelines outlined in DEC's DOW TOGS 3.1.5, *"Guidance for Dam Hazard Classification."* Roads and structures which might be impacted by the dam's failure are those present approximately 1,200 feet south of the dam, these being Cream Street and several homes along Cream Street. Beyond this area, no roads or structures are present within the reach of the stream corridor which would be significantly impacted by flows from a breach of the dam.

Cream Street is a low-use Town road that is unclassified according to the NYSDOT functional classification viewer. Section D.4 of TOGS 3.1.5, indicates that an impact to a roadway of this functional classification would result in the dam's hazard classification being Class "A," low hazard.

In order to determine the impact of flows on the downstream homes, three different flow scenarios were evaluated. Two flow scenarios involved failure of the dam, one being failure of the dam under normal flow conditions (termed "sunny day" failure) and the other being failure during the spillway design flood (termed "rainy day" failure). The third scenario evaluated involved determining the impact to the homes under the spillway design flood (SDF) but assuming the dam does not fail. For this scenario and for failure of the dam under the rainy day event, the SDF was determined assuming a design storm event applicable to the dam's current hazard classification of "B", this storm event being 150% of the 100-year storm for the area. A copy of the HEC-RAS model is contained in Appendix G.

For the rainy day failure scenario, the dam was failed at the peak of the storm event (a worst-case scenario) which, as presented in Section 4.0 of this report, is an event which will result in overtopping of the dam. Accordingly, the parameters used were based upon FERC parameters for an overtopping failure. The parameters selected were practical, but are considered somewhat conservative in order to provide a reasonable worst-case simulation of the dam's failure. The assumption made was that half of the masonry wall retaining the embankment on the left side of the spillway would fail as a result of the dam overtopping and that water would erode the downstream slope of the embankment, leading to complete failure of the dam. A breach width of 40 feet was assumed and that the breach would form fully to the bottom of the earthen embankment (near elevation 200.0 feet). Vertical side slopes to the breach were assumed due to the presence of the masonry wall at the upstream side of the embankment confining flow through the same. A relatively short failure time of 2 hours assumed for development of the full breach. The results of the rainy day failure scenario indicate that the peak outflow from the dam failure is 6,870 cubic feet per second (cfs).

The rainy day without failure scenario was modeled to compare the difference between the peak outflows with and without failure of the dam under this SDF event. The rainy day failure scenario has a peak outflow that occurs 1.5 hours after the peak from the watershed occurs. The flow from the watershed at that time was determined to be 3,670 cfs.

The sunny day failure was performed to model the peak outflow from the dam in the event of its failure under normal pool conditions. As with the rainy day failure model, FERC parameters applicable to a concrete dam were used. An overtopping failure of a section of the spillway was assumed. Of the twelve (12) masonry sections (blocks) present along the spillway, each of which is approximately 5.75 feet long, it was assumed that two of these blocks would fail and that failure would progress block by block to the bottom of the dam. Accordingly a vertical breach width equivalent to two masonry blocks was assumed (11.5 feet). Given that FERC guidelines recommend a range of failure time for a concrete dam from 0.1 to 0.5 hours, a failure time of 0.5 hours was assumed as the dam retains only a relatively modest height of water. The peak outflow from the assumed breach was estimated to be 1,345 cfs.

A HEC-RAS model of the downstream road crossing was prepared to assess the impact of the three flow scenarios on the downstream homes. The HEC-RAS model was run in a "steady" state feature, since there is limited attenuation of flood flows between the dam and Cream Street. The finished floor elevations of the four lowest-lying homes along Cream Street were determined by field survey and found to range from 197.30 for 16 Cream Street to 207.09 for 26 Cream Street.

Table 1 below provides a summary of the results of the flow/dam failure modeling under the three flow scenarios.

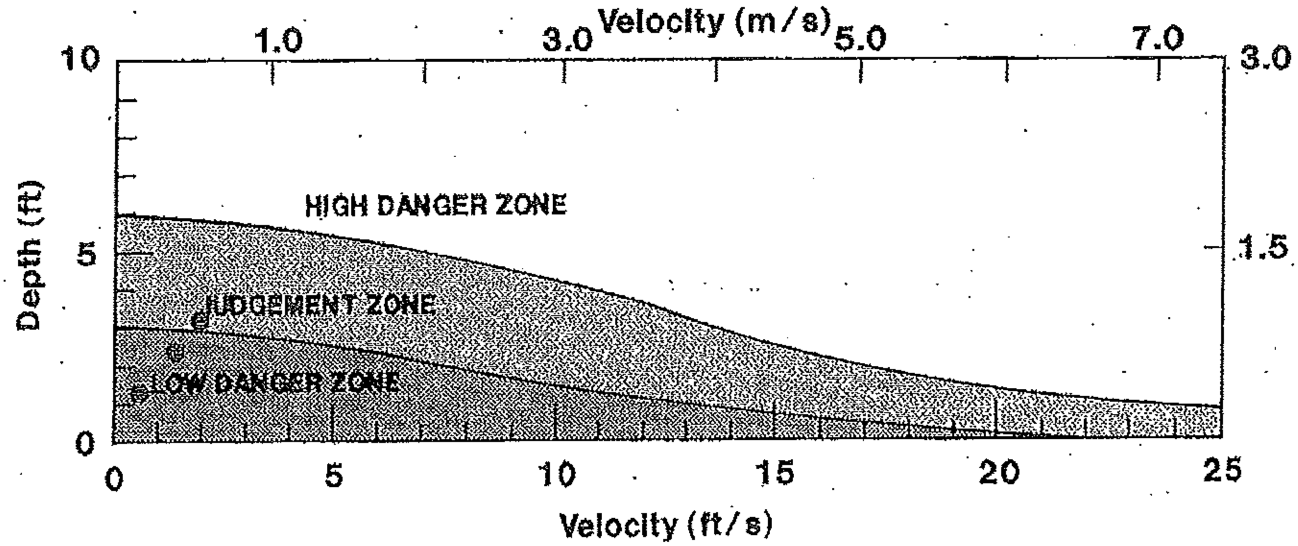
Table 1
Flow Simulation Results

Address Of Impacted Home	HEC-RAS Station	Finished Floor Elev. (FT.)	Estimated High Water Surface Elevation (FT.)		
			Rainy Day Failure (6,870 cfs)	Rainy Day No Failure (3,670 cfs)	Sunny Day Failure (1,345 cfs)
10 Cream St.	XS 790, STA 780	204.44	201.21	200.11	198.69
14 Cream St.	XS 790, STA 695	203.81	201.21	200.11	198.69
16 Cream St.	XS 505, STA 545	197.30	200.57	199.71	198.52
26 Cream St.	XS 885, STA 170	207.09	202.38	201.64	200.64

Table 1 indicates that under each of the flow scenarios, the only structure that would be impacted is that located at 16 Cream Street, which sits on a long driveway and is located very close to Fall Kill Creek. With the finished floor of this home being only 4 or so feet above the creek bed, the water level would rise under the flow scenarios from 1.22 feet to 3.27 feet above its first floor. Under the rainy day without failure scenario, the water level would be approximately 2.4 feet above the first floor and would rise an additional 0.86 feet if the dam were to fail. The velocity of flooding at 16 Cream Street, per the results of the HEC-RAS modeling, is 0.27 feet per second (fps) for the sunny day failure scenario, 1.33 fps for the rainy day scenario and 2.05 fps for the rainy day failure scenario.

In assessing the significance of the flow/dam failure simulations summarized above, Section D.3 of the DEC's TOGS 3.1.5 was reviewed. The guidance mentioned in TOGS 3.1.5 refers the reader to ACER 11 publication *"Downstream Hazard Classification Guidelines"*. Within this document is Figure 2 entitled *"Depth-Velocity Flood Danger Level Relationship for Houses Built on Foundations"*. This figure is presented on the following page and has superimposed on it the results of the flow/dam break simulations summarized above for the house located at 16 Cream Street. As shown on this figure, the sunny day failure and the rainy day without failure scenarios fall into the "low danger zone" while the rainy day failure scenario falls into the low end of the "judgment zone."

HIGH DANGER ZONE – Occupants of most houses are in danger from floodwater.
JUDGEMENT ZONE – Danger level is based upon engineering judgement.
LOW DANGER ZONE – Occupants of most houses are not seriously in danger from flood water.



Results of Flow/Dam Break Simulations for 16 Cream Street

Rainy Day with Failure: Velocity of Flow 2.05 fps w/ 3.27' of Water Above FF
Rainy Day, No Failure: Velocity of Flow 1.33 fps w/ 2.41' of Water Above FF
Sunny Day Failure: Velocity of Flow 0.72 fps w/1.22' of Water Above FF

Given that the occupants of 16 Cream Street would likely be evacuated as the creek levels would encroach upon the first floor level of their residence during a SDF event, their lives would likely not be in danger in the event of a dam failure. Accordingly, a hazard classification of "B" is still considered appropriate for Fall Kill Dam. This classification, by definition, is a *dam in which failure may result in damage to isolated homes, main highways, and minor railroads; may result in the interruption of important utilities... and/or is otherwise likely to pose the threat of personal injury and/or substantial economic loss or substantial environmental damage. Loss of human life is not expected.*

4.0 HYDROLOGIC & HYDRAULIC ASSESSMENT

4.1 Design Rainfall Analysis

With its intermediate hazard classification, the spillway design flood for Fall Kill Dam is 150% of the 100-year storm. The 100-year storm for the watershed is a Type III distribution and, from the Northeast Regional Climate Center, the storm equates to 8.29 inches of rainfall in 24-hours (See Appendix E).

4.2 Watershed Analysis

Delineation of the watershed tributary to Fall Kill Dam was performed using topographic information shown on the "Poughkeepsie, Hyde Park and Salt Point" USGS 7.5-minute quadrangle maps (Refer to Figure 1, Watershed Delineation Map). The total watershed area draining to the dam is relatively large and was computed to be 8,318 acres (13.3 square miles). The watershed was broken down into nineteen sub-areas, chosen to represent different stream segments that feed Fall Kill Creek. The modeling also took into account eight impoundments occurring upstream of the dam, the majority of which were small lakes or ponds, or low areas with impoundment potential formed by road crossings. Information on the outlet works of these impoundments was obtained from a combination of topographic survey and Google Earth. The reaches shown in HydroCAD and on Figure 1 are provided in HydroCAD to route flows in the model and to connect different portions of the watershed with each other.

Appendix F contains detailed soil data obtained from the USDA's Web Soil Survey. Soils with a dual rating (such as A/D) represent drained versus undrained conditions. For the purpose of developing a conservative watershed model for the SDF analysis, soils with a dual rating were considered as being Group "D", undrained. Land cover within the watershed varies and is based upon aerial imagery.

Appendix H, HydroCAD Modeling Reports, contains specific information related to each sub-watershed, including land use breakdown, curve number calculations and time of concentration calculations. The time of concentration paths are shown on Figure 1. Runoff from the watershed was evaluated using HydroCAD for the 100-year storm event. The basic runoff computation methodology employed by HydroCAD is *Technical Release 20* (*TR-20*) developed by the USDA Soil Conservation Service.

4.3 Spillway Capacity Analysis

As the basis for assessing the dam's spillway capacity and whether the dam would be overtopped during the spillway design flood (SDF), a limited topographic survey of Fall Kill Dam was performed. From this survey, it was determined that the spillway has an average crest elevation of 210.93 feet, a width of 69.3 feet, and a breadth of 25 feet. The stone masonry wall on the right side of the spillway has an average crest elevation of 215.05 feet and a length of 74 feet while the stone masonry wall to the left of the spillway has an average crest elevation of 216.13 feet and a length of 80 feet. The low points at the ends of the dam were not modeled as being present as it is recommended in Section 9.0 of this report that they be infilled to establish a uniform crest elevation. Also included in the HydroCAD models are the presence of the dike located a short distance upstream of the dam. Of the two HydroCAD models, one included the profile of the dike as it presently exists and one assumed that fill would be placed across the dike to a level which would prevent flow over the same. Per the topographic survey of the dike its existing condition was modeled with a length of 135 feet long and a crest elevation varying between 212.78 and 213.75 feet.

The 100-year storm event was simulated over the watershed and, consistent with Section 5.2 of DEC's publication *"Guidelines for Design of Dams"*, flow from the watershed was increased by 1.5 times before being routed over the spillway (per Link 1 in the HydroCAD model). The peak inflow into the impoundment during the 150% of the 100-year storm event was determined to be 4,114 cubic feet per second (cfs), occurring at hour 20.65 of the 24-hour design storm event. The peak outflow over the dam was determined to be 4,063 cfs, with a peak water surface elevation of 217.42 feet, occurring at hour 21.15. The peak water surface elevation simulated during the spillway design flood is 1.29 feet over the left (highest) side of the spillway assuming the dike has been infilled to prevent flow over the same. Hence the current requirement of Section 5.3 of DEC's publication *"Guidelines for Design of Dams"* that the spillways of dams have adequate spillway capacity to pass the design flood without overtopping is not met.

With the dike left in its current condition and flow allowed to occur over it, the peak water surface is elevation 217.19 feet. The left side of the dam would still be overtopped by 1.06 feet, indicating that the dike provides little benefit to the spillway capacity during the spillway design flood.

4.4 Storage Evacuation Calculations

DEC's current publication *"Guidelines for Design of Dams"* contains specific time requirements for lowering of the water level behind dams. Each requirement is identified below and the results of the evacuation calculations presented in Appendix I.

4.4.1 Spillway Evacuation

Section 6.3.2 of DEC's *"Guidelines for Design of Dams"* requires that for a single spillway, assuming no inflow, the spillway should have sufficient capacity to evacuate the storage between the maximum design high water and the spillway crest within 48 hours. The calculations indicate that the spillway is capable of discharging the storage between the maximum high water (217.42 feet) and the spillway crest (210.93 feet) in 1.9 hours, assuming no inflow. Accordingly, this evacuation criterion is met.

4.4.2 Low-Level Drain

Section 7.1 of DEC's *"Guidelines for Design of Dams"* requires that the low-level drain(s) have sufficient capacity to discharge 90 percent of the storage below the spillway crest within 14 days, assuming no inflow into the reservoir. The survey of the dam and past mapping indicates the presence of a 42-inch cast-iron drain which was assumed to be the dam's low-level drain. The 42-inch drain is approximately 100 feet long, with a downstream invert of 195.66 feet. For the purpose of modeling, it is assumed that the drain is flat. There is limited historical information regarding the depth of the reservoir, however, the Emergency Action Plan, prepared by Dutchess County, mentions that the reservoir at its deepest section is 20 feet and the depth near the edge of the reservoir is 2 feet. The storage of water below the normal pool was estimated based upon this information.

In order to drain 90 percent of the reservoir's storage, the water level would need to lower to elevation 201.75' (where the storage is 21 acre-feet). The calculations presented in Appendix I indicate that after 15 hours, the reservoir's level drops to elevation 201.75, hence, the low-level drain criterion of Section 7.1 of the DEC guidelines is met provided the drain is made functional.

5.0 SUBSURFACE INVESTIGATION PROGRAM

During the document review process, no information was recovered indicating that any subsurface investigation programs had been previously performed at the dam. However, it was noted in an undated document recovered during the document review process that the dam was to be founded upon sound bedrock which was present at depths ranging from 3 to 7 feet below the original ground surface. The overburden conditions present above bedrock are described as being "alternate strata of clay, gravel and boulders" [*sic*]. Due to the relatively limited size of the dam's embankments and access restrictions to the dike, no subsurface investigation was performed for this Engineering Assessment.

6.0 SUBSURFACE CONDITIONS

6.1 Embankment & Foundation Soils

No information was recovered regarding construction of the dam's earthen embankments or dike section. However, it is likely that they were to be constructed from locally available borrow materials. As such, the United States Department of Agriculture's (USDA) Web Soil Survey was consulted to determine the composition of the soils in close proximity to the dam. The USDA mapping indicates that these soils are composed of "loam", generally present to depths in excess of 6 feet below the ground surface. In several of the soil unit descriptions, it was noted that glacial till ("densic material") or bedrock was present at depths as shallow as 2 to 3 feet below the ground surface. Soils used to construct the embankment were therefore assumed to be composed of locally excavated and recompacted glacial till. As the dam was noted to be founded upon bedrock, it was assumed that bedrock was present beneath the spillway ("foundation soils"). However, it was assumed that glacial till was present beneath the dike's embankment.

6.2 Groundwater

For the purposes of this analysis, it has been assumed that the core wall present in the dike has no effect in lowering the line of seepage through its embankment. As such, the line of seepage through the embankment has been estimated using the methods outlined by L. Casagrande.

6.3 Soil Properties

Soil properties utilized in analyzing the stability of earthen embankments were estimated based upon past experience with similar soils. For the purposes of this analysis, it has been assumed that the soils possess only a frictional component of strength.

The soil properties utilized in this analysis are summarized in Table 2.

Table 2
Assumed Soil Strength Properties

Soil Layer	Internal Angle of Friction, ϕ (°)	Undrained Shear Strength, s_u (psf)
Embankment Soils	32	0
Foundation Soils (Dike)	32	0

7.0 STABILITY ANALYSIS

7.1 Embankment Stability

The stability of the embankment's downstream slopes was analyzed following procedures identified in the DEC publication *"Guidelines for Design of Dams"*. These guidelines reference the use of methods of analyses outlined in the Corps' publication *"EM 1110-2-1902, Slope Stability"*. Of the seven (7) load cases listed in this publication, only four (4) were deemed applicable for analysis of the dike and three (3) for the dam's earthen closure embankments. A description of each of these loading conditions, the slopes analyzed, and the minimum factors of safety recommended by the Corps for each load case are listed in Table 3.

Table 3
Required Factors of Safety

Case No.	Loading Condition	Pool Elev.	Slope Requiring Analysis	Required Factor of Safety
II	Long-Term with Steady Seepage	211.0′	Downstream	1.5
III	Maximum Surcharge Pool	217.42′	Downstream	1.4
IV	Rapid Drawdown	200.0′	Upstream	1.3
VII	Earthquake – Case II with Seismic Loading	As Noted Above	Upstream & Downstream	1.0

The stability of the embankments was analyzed using the computer program GeoStudio 2012, produced by GEO-SLOPE International, Ltd. Each of the load cases presented in Table 3 was analyzed for the dam's closure and dike's embankments along a cross-section where the inclination of the earthen embankments was visually estimated to be the greatest. As stacked stone masonry with a near vertical inclination retains the upstream side of the closure embankments, no analysis was performed for load cases affecting the upstream slopes.

For the purposes of this analysis, it was assumed that long-term ("drained") strength parameters were applicable. The line of seepage through the embankments has been estimated using the methods outlined by L. Casagrande. Although a concrete core wall is present within the dike, its concrete is cracked and highly deteriorated. In addition, seepage was observed emerging along the downstream toe of the dike section. Based upon these observations, it was assumed that the core wall has no effect on lowering the line of seepage through the dike's embankment.

The stability of the embankments was analyzed for Load Case VII under the maximum design earthquake (MDE), as defined in the Corps publication ER 1110-2-1806, *"Earthquake Design and Evaluation for Civil Works Projects"*. The MDE for a Class "B – Moderate Hazard" structure has been estimated as a seismic event with a 5-percent chance of exceedence in 50 years, or a return period of 1,000 years. From the USGS Interactive Hazard Deaggregation website, this event would produce a 0.20-second spectral acceleration of 0.106g at the bedrock surface. Using methods set forth in the Building Code of New York State which utilizes this bedrock surface acceleration and the Site Class applicable to the embankment's subsurface conditions (Site Class C), the approximate peak ground acceleration for the MDE event was estimated to equal 0.034g. A copy of the Interactive Hazard Deaggregation Graph is included in Appendix J along with calculations supporting the determination of the peak ground acceleration in accordance with the methods outlined in the Building Code.

7.2 Spillway Stability

The stability of the dam's spillway section was performed in accordance with the procedures presented in DEC's publication *"Guidelines for Design of Dams"*. The following four (4) load cases were analyzed for sliding and overturning stability of the structure:

- **Load Case I** – Normal Loading Condition – Water Surface at Normal Pool Level, Elevation 211.0 feet.
- **Load Case II** – Normal Loading Condition - Water Surface at Normal Pool Level plus Ice Load of 5,000 pounds per lineal foot (plf).
- **Load Case III** - Design Loading Condition - Water Surface at Spillway Design Flood Level, Elevation 216.43 feet.
- **Load Case IV** - Seismic Loading Condition - Normal Loading Condition with Water Surface at Normal Pool Level plus Pseudo-Static Earthquake Loading.

Elevations of the spillway section utilized in the analysis were based upon the limited topographic survey performed of the dam. Dimensions shown on the recovered historic field sketch appeared to be relatively consistent with those physically measured during the Initial Safety Inspection and, as such, were utilized in the stability analyses.

8.0 STABILITY RESULTS

8.1 Embankment Stability

The stability analyses were conducted along sections through the embankment and dikes where the inclination of the slopes was at its greatest. Using the soil properties and strength parameters previously discussed, the minimum calculated factors of safety under each loading condition are shown in Table 4 and Table 5. As the hydrologic and hydraulic analysis of the dam and its current spillway under the spillway design flood indicate that the dam will overtop, Load Case III was analyzed assuming that the embankment's crest would be armored to prevent its erosion during the short period it is overtopped during the spillway design flood. Computer generated output from GeoStudio is included in Appendix K.

Table 4
Computed Factors of Safety for Closure Embankment Slope Stability

Case No.	Loading Condition	Slope Analyzed	Calculated Factors of Safety	Min. Req. Factor of Safety
II	Long-Term with Steady Seepage	Downstream	1.70	1.5
III	Maximum Surcharge Pool	Downstream	1.70	1.4
VII	Earthquake - Load Case II with Seismic Loading	Downstream	1.87	1.0

Table 5
Computed Factors of Safety for Dike Slope Stability

Case No.	Loading Condition	Slope Analyzed	Calculated Factors of Safety	Min. Req. Factor of Safety
II	Long-Term with Steady Seepage	Downstream	1.57	1.5
III	Maximum Surcharge Pool	Downstream	1.41	1.4
IV	Rapid Drawdown	Upstream	0.96	1.3
VII	Earthquake - Load Case II with Seismic Loading	Upstream & Downstream	0.86 & 1.76	1.0

8.2 Spillway Stability

Table 6 on the following page presents a summary of the spillway's computed factors of safety against sliding along a horizontal plane taken through its downstream toe, or elevation 195.75 feet. Although it was noted in recovered documents that the dam was to be founded upon sound bedrock, for the purposes of this analysis it was assumed that the dam was constructed upon the original ground surface and was not keyed into the underlying soils. As noted during performance of the Initial Safety Inspection, little to no grout is present in any of the dam's masonry joints. However, it is noted that during the 1921 reconstruction work performed on the dam that the upstream face of the spillway was surfaced with concrete. Based upon the presence of this concrete facing, it has been assumed that the spillway will behave monolithically and is not able to experience sliding failure at each joint. Appendix L contains the calculations supporting the results of the stability calculations summarized below in Table 6.

Table 6
Computed Factors of Safety

Load Case	Load Case Description	Friction Factor of Safety		Location of Resultant Force
		Calculated	Required	
I	Normal Loading Condition - Water Surface at Normal Pool Level	1.82	1.50	Middle Third
II	Normal Loading Condition - Water Surface at Normal Pool Level plus Ice Load of 5,000 plf	1.09	1.25	Middle Half
III	Design Loading Condition - Water Surface at Spillway Design Flood Level	1.03	1.25	Middle Half
IV	Seismic Loading Condition - Normal Loading Condition w/ Water Surface at Normal Pool Level plus Pseudo-Static Earthquake Loading	1.37	1.00	Within Base

As shown in Table 6, only Load Cases I and IV have safety factors against sliding in excess of the minimum required values. Under both Load Cases II and III, the dam's spillway section does not meet the required factors of safety.

9.0 CONCLUSIONS & RECOMMENDATIONS

Documentation reviewed for the dam included several historic documents, past inspection reports compiled by DEC, and the dam's EAP prepared in March 2012. Based upon our review of this information and a Safety Inspection performed of the dam, it is our opinion that while the dam does not possess a rating of "unsafe" or "unsound", it does not meet all of DEC's dam safety criteria. The following subsections summarize the conditions requiring correction or further investigation and a schedule recommended for addressing the same should it be desired to maintain the dam in service rather than breaching it.

9.1 Stability Analysis

Structural stability calculations performed of the dam's spillway section determined that under Load Cases II and III, Ice Loading and Spillway Design Flood Conditions, respectively, the section does not meet the required minimum factors of safety against sliding. As such, it will be necessary to increase the spillway's resistance to sliding through the installation of drilled rock anchors. From preliminary calculations, it has been estimated that a minimum of three (3) rock anchors will be required to achieve the required factors of safety.

So as to prevent erosion of the dam's closure embankments during the short time the dam will be overtopped under the SDF event, their surface will need to be armored such as by the installation of articulated concrete blocks. At the earthen dike, its core wall should be extended to prevent its overtopping during the SDF event. Extension of the core wall will require removal of its upper deteriorated portion and capping of the remaining "sound" concrete with reinforced concrete. The mass of the new concrete cap and its connection to the core wall to remain will need to be carefully designed to be stable should the upstream slopes of the dike slough under Load Cases IV or VII (See Table 5). With such a design, modifications to the dike's upstream slope should not be necessary.

9.2 Spillway

Several pieces of the dam's stacked stone masonry were observed to be dislodged or missing. The absence or displacement of this masonry may lead to loss of support for the overlying courses and could potentially result in failure of the walls retaining the closure embankments. As such, it is recommended that any displaced or missing masonry be restored to their original location and the walls be repointed.

Apparent leakage through the dam's spillway section has been observed occurring through the joints between the dam's stacked stone masonry. This leakage extends from the first joint below the spillway crest to a point approximately halfway down the spillway's height. Along the interface of the stacked stone masonry and its concrete facing, grout should be injected under a low pressure to mitigate the seepage and joints on the downstream face of the spillway repointed.

Occasional pieces of deadfall are present along the crest of the spillway section. Its presence may serve as an impediment to flows over the spillway, and as such, it should be removed when present.

The concrete capping placed over the top of the spillway abutment/training walls was observed to be in relatively good condition. Occasional structural cracks were observed of the concrete, although at the time of inspection, the cracks were relatively tight with no loss of ground or differential movement having occurred. These cracks should be monitored during future inspections of the dam and if loss of ground observed, the cracks should be filled through the injection of a cement or epoxy grout.

9.3 Pipes & Gates

A valve house was formerly present on the crest of the right downstream embankment. The superstructure for this building is believed to have been demolished many years ago, although two (2) concrete slabs present are believed to cover the former structure's substructure. A wooden fold-out ruler was used to probe the area beneath the slabs and encountered soil at a depth of approximately one (1) foot below the top of slab. As no information was recovered regarding the valve house, it is unclear as to what was located within the same and what work, if any, was done at the time of the superstructure's demolition. It is recommended that the slabs be temporarily moved to allow for inspection and observation of the former valve house's substructure.

A 42-inch diameter cast-iron pipe discharges into Fall Kill creek at a point approximately 100 feet downstream of the former valve house. The purpose of this pipe is unknown, although it is suspected to be the low-level drain for the reservoir. The pipe's sluice gate should be restored to a working condition or replaced to allow for drawdown/evacuation of the reservoir. The condition of the pipe itself as well as the head/entrance conditions at its upstream end should be investigated to ascertain it is structurally sound and capable of conveying flows through it.

9.4 Embankment

Portions of the wall retaining the right closure embankment were observed to be leaning from vertical by several degrees towards the reservoir. Over time, this condition may continue to deteriorate, particularly after repeated exposure to freeze-thaw cycles. It is recommended that the wall be restored to a near vertical condition by temporary excavation of the wall's backfill to allow for restoration of the stone masonry to its original location and inclination. Once this is complete, the joints should be repointed.

Numerous large diameter trees and tree stumps present across the dike and the dam's closure embankments should be removed. Fill should be placed on the closure embankments to establish a uniform crest elevation and the surface of the embankments armored to prevent erosion during overtopping of the dam. The dike's concrete core wall should be extended and its downstream slope vegetated to inhibit its erosion. This slope should be maintained in a groomed condition to facilitate its visual inspection for signs of seepage, erosion, animal burrows, and slope instability. Field grass and weeds may remain in place but should be mowed and/or weed-whacked on a regular basis. Any brush which is present should be removed. The growth should be maintained to a height on the order of three (3) inches or less. Where barren soil is exposed by this work, it should be seeded and mulched to promote establishment of a vegetative cover.

No animal burrows were observed at the time of inspection. In the event that they are observed during future inspections, rodents and animal burrows/nests should be dealt with as follows. Two acceptable repair techniques for rodent holes are mud-packing and excavation/backfilling. Mud-packing is an appropriate technique for use between construction cycles and can be accomplished by maintenance staff as ordinary maintenance of the dam. Mud-packing involves placing one or two lengths of metal stove or vent pipe vertically over the entrance of the den with a tight seal between the pipe and den. A mud-pack mixture should then be poured into the pipe until the burrow and pipe are filled with the earth-water mixture. The pipe should then be removed and additional dry earth tamped into the entrance. The mud-pack mixture should be made by adding water to a 90 percent earth and 10 percent cement mixture until a slurry of thin cement-like material is attained. All entrances should be plugged with well-compacted earth and vegetation re-established. Where large burrows or extensive rodent tunneling resumes, the damaged areas should be excavated down to competent soil and repaired as previously described. Once repairs are completed, the area should be reseeded or resurfaced with rip rap.

9.5 Concrete Core Wall

The dike's concrete core wall was observed to be in poor to fair condition, with moderate to significant deterioration noted of that portion of the core wall which was exposed. As the crest of the dike is to be elevated to prevent overtopping under the spillway design flood event, it is recommended that the core wall also be extended as previously recommended in Section 9.1

9.6 Recommended Schedule of Work

Each of the above described deficiencies should be corrected in order to bring the dam into total compliance with DEC regulations. Rudimentary remedial actions such as the capture and relocation of animals burrowing into the dam do not require a permit and can be performed by maintenance personnel without incurring significant costs.

Other remedial work required to bring the dam into regulatory compliance carries with it the preparation of plans and specifications for addressing the same. Adequate time for the review of the plans and specifications and for permitting of the work by DEC and other regulatory agencies must be provided as well as for Dutchess County to be provided a budget estimate for the remedial work and to make funds available for its completion. With these considerations and our opinion that the dam does not have a Condition Rating of "unsafe" or "unsound", the schedule shown in Table 7 on the following page is recommended to address the deficiencies.

Table 7
Schedule for Addressing Deficiencies

Work Item	Schedule
Preparation of Specifications and Procedures for Remedial Work	2017
Submit Project Specifications & Procedures to DEC - Allow 3 months for review	2017
Advertise & Award Remedial Work Contract	2018
Completion of Work	2019

10.0 CLOSURE

This Engineering Assessment of Fall Kill Dam has been based upon the results of an Initial Safety Inspection, embankment slope and structural stability analyses, and hydraulic and hydrologic analyses. Several deficiencies have been identified which prevent it from being in compliance with DEC regulations. Recommendations and a proposed schedule for implementing the same have been provided in this Engineering Assessment.

Respectfully Submitted,

C.T. MALE ASSOCIATES

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Compilation of Codes, Rules and Regulations of the State of New York Currentness

Title 6. Department of Environmental Conservation

Chapter X. Division of Water Resources

Subchapter A. General

Section 673.4. Permit Requirements for Dams

6 NY ADC 673.4 COMPILATION OF CODES, RULES AND REGULATIONS OF THE STATE OF NEW YORK *(Approx. 3 pages)*

Article 1. Miscellaneous Rules

Part 673. Dam Safety Regulations (Refs & Annos)

6 NYCRR 673.4

Section 673.4. Permit Requirements for Dams

Regulations implementing the permit requirements for dams are set forth, without limitation, in Part 608 of 6 NYCRR, "Use and Protection of Waters." Nothing in this Part 673, or any order, notice or recommendation issued pursuant to this Part, shall be construed to relieve any dam owner of any obligation to obtain permits pursuant to Part 608 of this Title or of any other requirement of law, including but not limited to those of the Freshwater Wetlands Act, the Fish and Wildlife Law, the Federal Power Act, and the State Environmental Quality Review Act, unless otherwise specified therein.

Credits

Sec. filed Dec. 10, 1985 eff. 30 days after filing. Repealed and new filed July 31, 2009 eff. Aug. 19, 2009.

Current with amendments included in the New York State Register, XXXVIII, Issue 46 dated November 16, 2016.

6 NYCRR 673.4, 6 NY ADC 673.4

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